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Comparison of rapid load pile testing of driven and CFA piles installed in high OCR clay

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Abstract

The current analysis of rapid load tests (RLT) such as Statnamic is normally based upon empirical correlations with static pile tests in similar soils. In certain soil types, such as clays, the number of case studies used to develop analysis and allow selection of appropriate rate effect correction are limited. Due to these limitations, no distinction is made in the selection of correction factors between pile type and pile installation techniques. In clay soils it is well known that driven piles may have a significantly enhanced capacity over cast in situ piles of similar cross-section. To test the effect of pile installation techniques on RLT analysis, RLT testing and static testing were undertaken on precast driven concrete piles and cast in situ CFA piles installed in high plasticity London Clay. The results show that the installation technique does not appear to affect the magnitude of the rate effects, provided modifications are made to the analysis to account for the previously reported differences in static capacity between different installation techniques. Based upon the findings, it is suggested that a distinction should be made in RLT analysis between pile type and installation techniques, and for existing analysis techniques, further case studies based on rate correction parameters are required, especially in clay soils.

Keywords: Rapid load testing; Driven piles; CFA piles; Rate effects; Clay (IGC: A01, E04, K07)

1. Introduction

The analysis of rapid load pile testing (RLT) such as Statnamic (Middendorp, 2000) is currently heavily dependent on the use of empirically derived damping or rate effect parameters to correct for the viscous effects in soil at elevated strain rates. Recent developments to RLT analysis include the selection of damping and correction parameters based upon soil type (Paikowsky, 2004; Middendorp et al., 2008) and measureable properties such as Atterberg limits in clays (Powell and Brown, 2006).

Currently the rate effect parameters are derived from a direct comparison of the RLT load-settlement behaviour with that of a static pile test on the same pile or an identical pile installed in close proximity. Alternatively, the parameters may have their origin in high strain rate laboratory element testing (for example Schmuker, 2005). Unfortunately, in the former case there is a lack of high quality case study data upon which to confidently specify rate effect parameters especially in fine grained soils such as clays or silts. This has led to reluctance by some authors

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Nomenclature

A	pile shaft or base surface area
a	pile acceleration
F_{base}	pile base resistance
F_{shaft}	shaft friction resistance
F_{STN}	measured rapid load test resistance
F_u	derived static equivalent capacity
$F_{u,design}$	design static capacity
I_v	viscosity index
LI	liquidity index
LL	liquid limit
M	pile mass
m and n	Soil dependant rate parameters
N_q	bearing capacity factor
PI	plasticity index
s_u	undrained shear strength
Δv	relative velocity or penetration rate of pile and soil
v_0	reference velocity
$v_{\min}$	lowest velocity used in derivation of rate parameters
α	adhesion factor
γ	bulk density
δ_h	pile-head settlement
μ	UPM correction or reduction factor
τ_{lim}	limiting elevated rate shaft friction
τ_s	static shaft friction

to specify correction parameters in clays (McVay et al., 2003). This may result in a lack of end-user confidence in test results determined in fine grained soils and ultimately limit further development. Determining rate effect parameters from laboratory element testing is appealing from the point of view of material consistency and control of testing conditions, but, historically, testing has been undertaken at strain rates much lower than those experienced in full scale RLT (Leinenkugel, 1976; Sheahan et al., 1996; Katti et al., 2003). Rate effect analysis techniques developed on this basis (Krieg and Goldscheider, 1998; Schmuker, 2005) may then not be appropriate when applied to RLT tests.

Although the effect of soil type on RLT analysis appears to have been recognised (Paikowsky, 2004; Powell and Brown, 2006; Middendorp et al., 2008), the effects of pile type and installation techniques has had limited investigation. For instance, in clay soils a driven pile (displacement) is likely to have relatively higher static ultimate capacity than a pile of similar cross section and length installed by boring techniques and cast in situ (non-displacement). The effect on pile shaft capacity of the method of installation is well documented with bored piles displaying approximately 70% of a driven pile's shaft capacity (Fleming et al., 2009). This is also reflected in the higher adhesions factors used in total stress design for driven piles (Weltman and Healy, 1978). It is not currently clear if an associated increase in pile resistance would be measured during an RLT test, which would therefore allow the use of the same correction parameters for both displacement and non-displacement piles.

Due to the tendency for increased static capacity of displacement piles over non-displacement piles in clay, it is necessary to investigate this effect on both RLT analysis and parameter selection. For instance, the technique proposed by Schmuker (Krieg and Goldscheider, 1998; Schmuker, 2005; Middendorp et al., 2008) has its origins in low strain rate laboratory element testing, which cannot easily replicate pile-soil interface behaviour, complicated variations in situ effective stress or the effects of the high soil strain levels encountered during pile driving. The analysis method proposed by Powell and Brown (2006) and Brown and Hyde (2008) derives the majority of its soil dependant rate parameters from both back analysis of RLT field studies on non-displacement cast in situ piles and high strain rate (push-in) probing tests (Brown, 2008). Although the probe tests are a "displacement" type event, they do not reflect the "restrike" approach of RLT testing, where the pile is tested some time after installation.

In order to investigate the effect of pile installation technique and increase the available case study information for RLT in fine grained soils, a series of driven precast piles were installed at a research site underlain by Quaternary London Clay. The results of RLT and static testing of these piles was compared with the results from testing cast in situ continuous flight auger (CFA) piles installed at the same site. The pile testing described in this paper was undertaken as part of an industry led research project (RaPPER, Rapid Pile Performance Evaluation Resource) which was designed to give guidance on testing piles for re-use (Butcher et al., 2006) and the applicability of different pile testing methods in different soil types.

2. Field study site

The study site is located at Lodge Hill Camp, Chattenden, Kent in the UK and is underlain by London Clay to a depth in excess of 35 m. The upper 4 m is typically weathered/desiccated brown London clay (OCR 50–24) which overlays unweathered blue clay of very high plasticity. The undrained shear strength in the upper 10 m gradually increases with an average shear strength of 100 kPa (average OCR 18). The plasticity index, $PI = 60\%$ in upper 10 m, rises to 63% for 10–15 m. The average moisture content in the upper 15 m was 29% and the bulk density, γ, was 19.4 kN/m^3. The water table was at approximately 1 m depth. The soil strength and characterisation data are shown summarised in Fig. 1. The site has been used extensively in recent times for pile behaviour testing (Skinner et al., 2003, Powell and Skinner, 2006) and more specifically to investigate RLT testing in clays (Powell and Brown, 2006, Brown and Powell, in press).

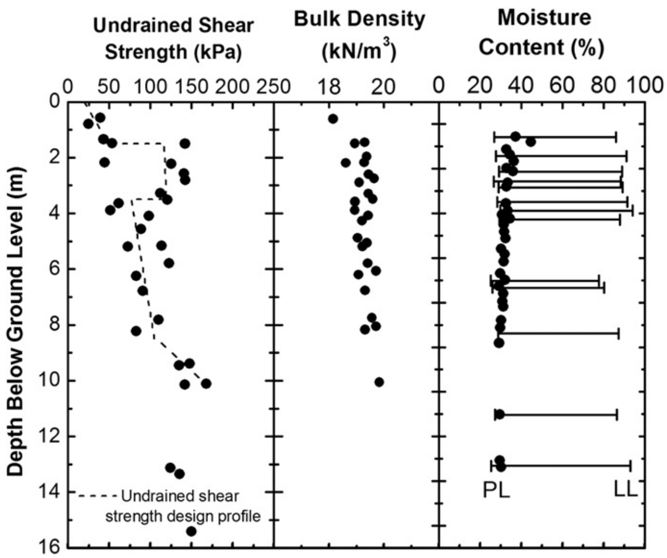

Fig. 1. Typical soil characteristics for the Chattenden site (Brown and Powell, in press).

3. Piles and testing regime

Pile testing was undertaken on both driven precast piles and cast in situ continuous flight auger (CFA) piles installed at the site. The precast driven piles were 11.0 m long (non-segmental), driven to a depth of 10 mBGL and had a square cross section of 275 mm × 275 mm. The cast in situ 450 mm diameter continuous flight auger piles were installed to a depth of 10.8 mBGL with an effective length of 9.667 m due to extension casing installation. The CFA piles were extended above ground at the time of casting by adding an 11 mm thick steel casing of 500 mm diameter filled with concrete. Some excavation locally around the head of the pile was required to allow this to occur. The design or characteristic static load capacity ($F_{u,design}$) of both types of pile was approximately 1000 kN.

In total four precast driven piles and seven CFA piles were tested in the study. For each pile type "identical" piles were installed and reserved for testing by a specific technique; more specifically, one pile was subject exclusively to RLT tests and was compared with the static test results for an adjacent pile rather than carrying out both tests on one pile. The pile types and the tests they were subjected to are shown in Tables 1 and 2. Note that where tests are made up of multiple cycles, the settlements reported (δ_h) are cumulative for all of the cycles.

3.1. Static pile testing

Static pile tests were performed using a hydraulic jack reacting against a frame restrained by anchor piles with loads measured directly using a calibrated load cell. The test procedure used complied with the ICE Specification for Piling and Embedded Retaining Walls (SPERW) (Institution of Civil Engineers, 2007).

Table 1
Summary of pile testing for the driven precast piles.

Pile	Cycles	Test type	Max. applied load (kN)	δ_h at max. Load (mm)	Max. δ_h during test (mm)
S1	6	RLT	2405	40.79	99.85
S2	6	RLT	2521	39.86	88.36
TP1	3	MLT	1124	5.67	18.02
	2	[a]CRP	950	18.70	41.54
		[b]CRP(H)	1043	23.47	41.54
TP2	2	CRP	1136	5.23	39.64
		CRP(H)	1212	7.99	39.64

[a]CRP test following directly from ML testing.
[b]CRP(H) refers to a phases of increased penetration rate during a constant rate of penetration test at standard rate (see Fig. 2).

Table 2
Summary of pile testing for the CFA piles.

Pile	Cycles	Test type	Max. applied load (kN)	δ_h at max. Load (mm)	Max. δ_h during test (mm)
CS1	4	RLT	3028	14.71	19.71
DC1	4	RLT	3825	13.88	25.15
R1	7	RLT	3976	19.63	43.44
MC1	1	MLT	1003	3.62	44.37
MC2		MLT	1128	6.66	29.19
		CRP[a]	570	28.61	46.13
		CRP(H)[b]	832	32.33	46.13
MC3		CRP	1120	4.76	40.59
		CRP(H)	1215	8.49	40.59
MC4		CRP	1098	4.15	89.47
		CRP(H)	1172	9.44	89.47

[a]CRP test following directly from ML testing.
[b]CRP(H) refers to a phases of increased penetration rate during a constant rate of penetration test at standard rate (see Fig. 2).

Two driven precast piles were tested to prove ultimate loads, one with a maintained incremental load procedure (ML) (TP1) followed by a constant rate of penetration stage (CRP). The second pile (TP2) was tested just using CRP procedures (Fig. 2). SPERW (Institution of Civil Engineers, 2007) defines the ultimate load in ML testing as the maximum load that can be applied whilst achieving a specified settlement criteria, and in CRP testing, it is defined as the maximum load prior to the point where loads have been reducing for 10 mm of settlement (or settlement equivalent to 15% of the pile diameter).

The test procedure employed for the ML test on the driven pile TP1 was to increase the loads in 125 kN increments with unload/reload cycles at 500, 750 and 1000 kN (0.5, 0.75, 1.0 × $F_{u,\ design}$). The CRP tests for piles TP1 and 2 were undertaken at an average constant rate (Δv) of 0.01 mm/s until a peak load had been reached. At this point, the rate of loading was increased to the safe maximum of the system, resulting in typical average settlement rates of 0.103 mm/s (referred to as CRP(H) and labelled as C and D in Fig. 2), for a short period to assess the effect of the rate of loading on the ultimate capacity. A similar approach to static testing was adopted for the CFA piles, as summarised in Table 2.

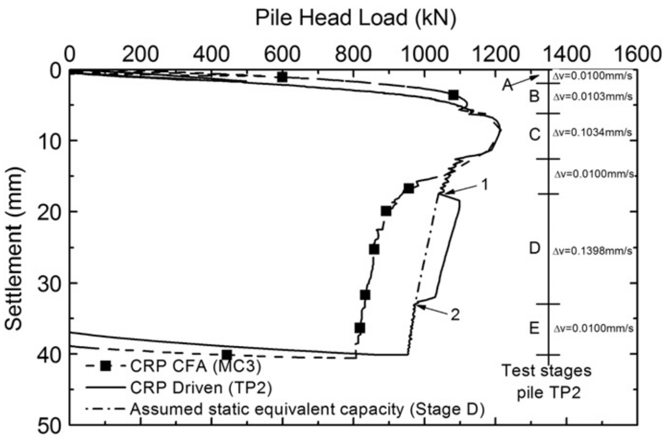

Fig. 2. Comparison of static CRP testing for a driven precast pile (TP2) and a CFA cast in situ pile (MC3). Stages A–E refer to variation in pile penetration rate for pile TP2 only (see Table 3). Labelling of the stages for pile MC3 have been omitted for clarity.

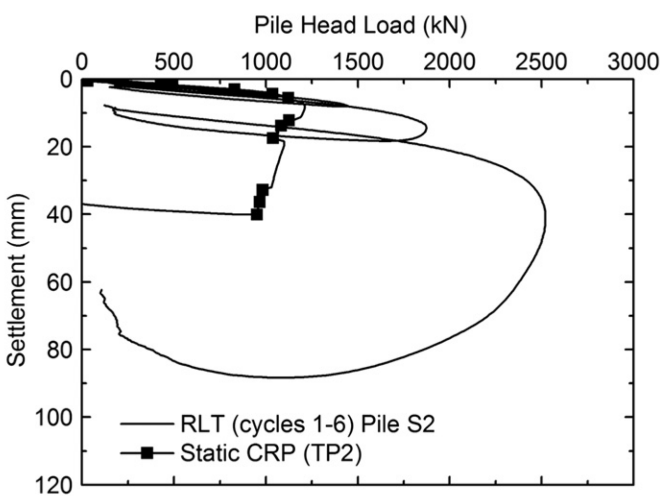

Fig. 3. Comparison of RLT load cycles (S2) with CRP static testing (TP2) for the precast driven piles.

3.2. Rapid load pile testing

Rapid load testing (RLT) consisted of Statnamic testing undertaken using a 4 MN rig with a hydraulic catch mechanism as described by Middendorp (2000). For both types of pile, several cycles of RLT loading were applied in quick succession on the same pile with each cycle increasing in magnitude. The duration between load cycles was controlled by the time to refuel the Statnamic device which typically took 15 to 30 min. The selection of load cycle magnitude generally followed the pattern of 0.75, 1.0, 1.5, 1.7, 2.5 times the static design load for the driven precast piles (Tables 1 and 2, Fig. 3). The selection of load cycle magnitude was less systematic for the CFA piles and was varied as each pile was tested to produce significant settlement (Fig. 4). For these piles the RLT load cycles

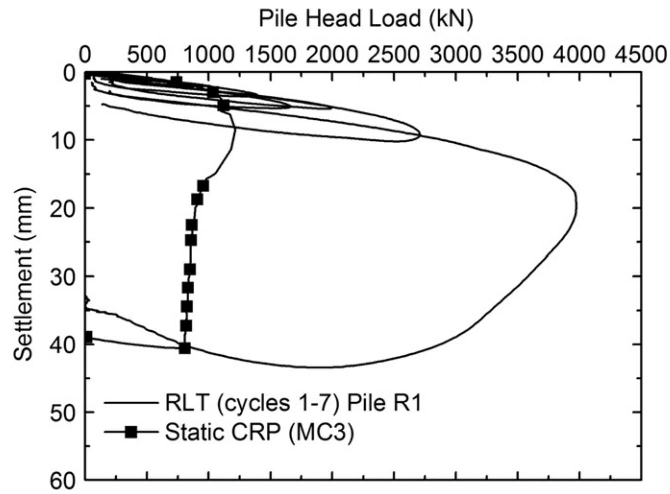

Fig. 4. Comparison of RLT load cycles (R1) with CRP static testing (MC3) for the cast in situ CFA piles.

varied approximately between 0.87 and 4 times the static design capacity.

4. Discussion of results

4.1. Results of static testing

Typical results of the static CRP pile testing are compared for the CFA cast in situ pile MC3 and the driven pile TP2 in Fig. 2 with key results summarised in Table 3. For reference purposes, certain key features or stages on the graphs are referred to using the letters A–E. Stage A refers to the first cycle of standard rate CRP testing to approximately half of the static design load. At this point, the settlement of the CFA pile was approximately half that of the driven pile, which is to be expected based upon the reduced cross section of the precast pile. Stage B indicates the initial peak bearing capacity reached for both piles at standard settlement rates, which is of a very similar magnitude for both pile types (Table 3) and highlights the enhancement of pile capacity due to the difference in installation techniques between the two pile types. Pile settlement is also reported at 495 kN in Table 3 which reflects working load settlements with 495 kN selected as a common load level encountered in Stage A of the two CRP tests reported. Assuming a simple total stress analysis for shaft friction resistance (F_{shaft}) where

$$F_{shaft} = \alpha \times s_u \times A_{shaft} \quad (1)$$

and the pile base resistance (F_{base})

$$F_{base} = N_q \times s_u \times A_{base} \quad (2)$$

where N_q is assumed to be 9 and A_{shaft} and A_{base} refer to the surface area of the pile shaft and the area of the pile base respectively. Back analysis was undertaken to obtain the average shaft resistance by subtracting the calculated pile tip force (Eq. (2)) from the peak static pile capacity measured during Stage B (Table 3) of static pile testing. The remaining force was assumed to be due to skin resistance which was

Table 3
Comparison of static tests on the driven and CFA piles.

Pile	Stage[a]	Test type	Max. applied load (kN)	δ_h at max. Load (mm)[c]	δ_h at 495 kN (mm)[c]	Working load stiffness (kN/mm)	Average penetration rate, Δv (mm/s)
MC3	A	CRP	540	0.86	0.74	669	0.0102
	B	CRP	1120	4.53	0.61	814	0.0096
	C	CRP(H)	1215	8.26	–	–	0.1676[b] (0.2146)[b]
	E	CRP					0.0120
TP2	A	CRP	497	1.65	1.64	302	0.0100
	B	CRP	1138	5.23	1.34	371	0.0103
	C	CRP(H)	1212	7.99	–	–	0.1034[b] (0.1450)[b]
	D	CRP(H)	1099	18.63	–	–	0.1398
	E	CRP					0.0100

[a]The stages correspond to the labels on Fig. 2.
[b]Unable to maintain a constant penetration rate, peak shown in parenthesis.
[c]Pile settlements are for the cycle under consideration only (see Tables 1 and 2) and have been reset to remove the effect of earlier cycles.

divided by the pile shaft area to determine the average unit skin friction. For analysis purposes, the CFA piles were assumed to be of cylindrical cross-section. The design profile of undrained shear strength shown in Fig. 1. was then used to calculate a value of α based on Eq. (1) assuming a constant value of α over the shaft length. Back analysis of the standard rate static load test data gives an adhesion factor $\alpha = 0.98$ (average unit skin friction $= 95\,\mathrm{kN/m^2}$) for the driven pile and 0.73 (average unit skin friction $= 69\,\mathrm{kN/m^2}$) for the CFA pile at peak capacity i.e. an adhesion factor ratio of 0.75 ($=0.73/0.98$) between the driven and cast in situ piles, which is slightly lower than the ratio of 0.8 suggested by Fleming et al. (2009). The increased adhesion factor for driven piles is consistent with the findings of Weltman and Healy (1978) and Bond and Jardine (1991).

Stage C shows the effect of the increased settlement rate associated with the CRP(H) test on the two pile types. The high settlement rate peak strength is almost identical for the two pile types. As the peak capacity at the standard rate was very similar (Stage B) for the two piles (Table 3) this would appear to show that the enhancement of capacity with increased settlement rate is also similar, suggesting that the peak magnitude of rate effect is unaffected by the installation technique (over the range of penetration rates investigated). If rate enhancement of the pile tip component is ignored (Brown, 2004), this suggests an average increase in shear strength on the shaft from 95 kPa to 103 kPa. Fixing the undrained shear strength at the initial in situ values the adhesion factor increases to 1.05 ($\alpha = 0.98$ at standard rate) and 0.81 ($\alpha = 0.73$ at standard rate) for the driven and CFA piles respectively.

As the settlement rate varies slightly between the CRP(H) on the driven and CFA piles, it is useful to introduce a relationship that allows the representation of the rate effect whilst normalising for the pile settlement rate or pile velocity. The approach shown in Eq. (3) was developed by Randolph (2003) to allow the representation of pile shaft capacity enhancement during pile driving:

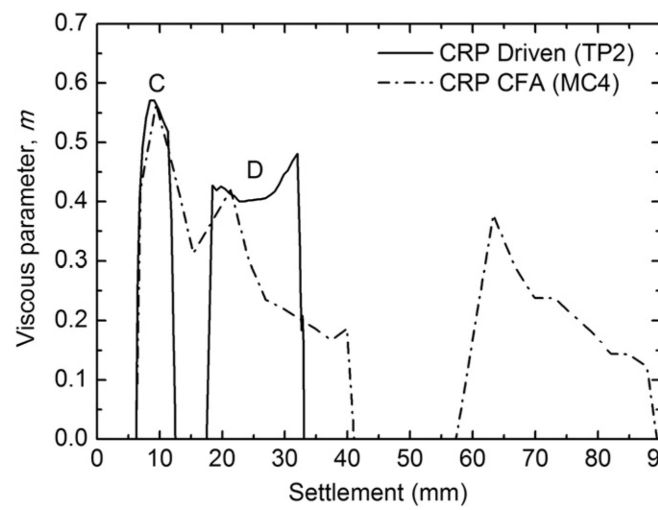

Fig. 5. Comparison of viscous rate parameters from elevated rate CRP(H) testing of the driven (TP2) and CFA piles (MC4).

$$\tau_{\lim} = \tau_s \left[1 + m \left(\frac{\Delta v}{v_0} \right)^n \right] \tag{3}$$

where τ_{lim} is the limiting elevated rate shaft friction, τ_s the static shaft friction, m and n are viscous parameters and Δv is the relative pile-soil velocity, normalised by v_0 (taken as 1 m/s). For clay soils, n is normally set to 0.2. To compare the rate effect, the viscous parameter m has been back calculated using Eq. (3), which normalises any variation in settlement rates and static pile capacity. The resulting variation of m for the two pile types is shown in Fig. 5. Comparison of the driven pile (TP2) has been made with pile CFA MC4 as the CRP(H) tests undertaken on this pile occurred at similar settlement levels to those of the driven pile. The process used to back calculate m can be understood by considering Stage D shown in Fig. 2 for pile TP2. In this case, τ_{lim} is the unit skin friction measured during stage D at the elevated rate of penetration (Δv). The magnitude of τ_s is determined by calculating the shaft resistance for the equivalent static or standard rate (v_0) test during this phase. This is achieved by considering the static shaft resistance just before the rate is increased (point 1, Fig. 2) to that associated with τ_{lim} in Stage D and at the end of Stage D when the rate of penetration again returns to the standard rate (point 2). Between these two points an equivalent static pile resistance variation is assumed, as shown in Fig. 2. This is used in turn to determine the assumed static pile resistance variation (τ_s), which is used in the back calculation of m.

At the low settlements associated with peak pile capacity (Stage C), the value of m is identical for both types of pile. Again, as settlement increases (Stage D), the CRP(H) tests show similar initial values of m although they appear to reduce rapidly for the CFA pile. This appears to suggest that the viscous rate effects are initially the same for the two types of pile installation and the rate effect itself is not affected by pile type or pile installation technique. Although the behaviour is initially similar, the viscous parameter reduces significantly after the initial peak with increasing settlement or strain for the CFA pile. This may purely be an artefact of the testing and/or analysis employed or it may reflect the fast shearing and level of strain the soil around the driven pile has experienced during installation. Tika et al. (1992) showed that slow shearing of London Clay after a phase of fast shearing (which may be compared here to pile driving) showed distinctive initial peak resistance well above that associated with low rate soil-soil residual friction angles which reduced gradually with increasing settlement to resistance associated with low rate residual soil-interface friction angles. The significant degradation in m with settlement for the CFA pile may be caused by continuing preferred orientation of the platy clay particles in London Clay to allow sliding shear along a highly polished residual shear surface as described by Tika et al. (1996). This surface would already have been fully formed for the driven pile.

On reducing the settlement rate to the standard rates associated with stage A and B (Fig. 2), both pile installation types show strain softening behaviour, with this being greater for the CFA pile where the ultimate bearing capacity at the end of loading (Stage E) is 72% of the low settlement rate (CRP) peak capacity. The strain softening behaviour is not as marked for the driven pile, with ultimate capacity being 84% of the peak load from CRP. This reduced degradation is likely to be due to the lower component of the tip capacity and the preferred orientation of platy clay particles to form well defined shear planes for the driven pile. This is highlighted in Tables 1 and 2 for piles TP1 (Driven) and MC2 (CFA) where peak capacities measured by CRP following MLT are significantly reduced with the effect being significantly greater for the CFA pile (MC2).

4.2. Results of rapid load pile testing

The results of RLT loading on the CFA piles and driven piles are shown in Figs. 3 and 4. It is apparent from the figures that significantly larger loads need to be applied to the piles during RLT loading to achieve equivalent or greater settlements created during static loading and especially to fully mobilise the piles. For example, in Fig. 3, the peak applied RLT load for cycle 6 which causes the largest settlement for the driven piles is 2521 kN (S2). This is 2.22 and 2.56 times the standard rate peak (Stage B) and ultimate (Stage E) capacities determined during the CRP static test. The maximum settlement rate of the pile during the RLT test was 2620 mm/s, which compares to 0.01 mm/s during the CRP test. By comparison, to achieve significant settlement for the CFA piles (Fig. 4), a load of 3976 kN (R1) was applied, which is 3.55 and 4.35 times the standard rate peak (Stage B) and the ultimate (Stage E) capacities determined during the CRP test (MC2). At peak loads, this would suggest that the apparent rate effects for the driven pile are approximately 72% of those for the CFA pile, although it is difficult to make direct comparison as the maximum pile settlement rate (1293 mm/s) for the CFA pile was approximately half that during RLT of the driven pile (2620 mm/s).

4.3. Rapid load test analysis

Several methods have been developed to analyse RLT tests which aim to derive the static equivalent load-settlement behaviour through removal of both inertial and soil dependant rate effects. These are commonly referred to as the unloading point method (UPM, Middendorp et al., 1992, Middendorp, 2000) and the Schmuker method (Schmuker, 2005, Middendorp et al., 2008). Brown and Hyde (2008) proposed a non-linear velocity dependant technique (referred here simply as the Brown method) based upon Eq. (3) of the form:

$$F_u = \frac{F_{STN} - Ma}{1 + (F_{STN}/F_{STN\ peak})m(\Delta v/v_0)^n - (F_{STN}/F_{STN\ peak})m(v_{\min}/v_0)^n} \quad (4)$$

where F_u is the derived static pile resistance, F_{STN} is the measured Statnamic load where the subscript peak denotes the peak load measured during the RLT test, Ma is the pile inertia, Δv is the pile's velocity relative to the soil and v_{min} is the velocity of the static CRP pile test used to define the soil specific rate parameters m and n. The parameter n is normally set to a value of 0.2 for clay soils (Randolph and Deeks, 1992). It has been proposed that the value of m may be linked to soil plasticity (Brown and Powell, 2006) by the relationship:

$$m = 0.03PI(\%) + 0.5 \quad (5)$$

This relationship has been shown to be valid in clay soils from low to very high plasticity (PI=7–68) for RLT loading events with velocities varying between 200 to 2000 mm/s (Brown and Powell, 2006, in press). The relationship itself has its origins in several studies with penetration velocities covering a wider range of velocities between 0.01 to 2000 mm/s (Brown and Powell, 2006).

Schmuker (2005) proposed a soil specific analysis technique which relies on the selection of a soil viscosity index parameter $I_{v\alpha}$.

$$F_u = (F_{STN} - Ma) \cdot (0.02\ \text{mm/min}/\Delta v)^{Iv\alpha} \qquad (6)$$

The viscosity parameter is related to a simple description of the soil as shown in Table 4. The viscosity index parameter has previously been determined for a range of soils including silts, clays and even organic soils based upon low strain rate multi-axial testing (Leinenkugel, 1976), triaxial testing and CRP pile testing (Krieg and Goldscheider, 1998). In these original studies, the typical strain or penetration rates studied were significantly lower than those encountered during RLT testing, which typically vary between 100 and 2000 mm/s. For example, in the study by Krieg and Goldscheider (1998), the pile penetration rates were only varied between 0.02 and 2.0 mm/minute. The parameters proposed by Schmuker (2005) do not vary significantly from those originally proposed and have only been applied to RLT testing in limited number of cases. Middendorp et al. (2008) present an example of the analysis technique applied to RLT testing of a single pile installed in low plasticity clay intermixed with sand layers where pile velocities reached 320 mm/s. The results appear to show a tendency for the analysis technique to under correct the rate effect during RLT testing which may be a result of the low velocity origins of the approach (Brown and Powell, in press).

The unloading point method is described in detail by Middendorp et al. (1992). Unfortunately, when this technique is applied to piles installed in fine grained soils, there is a tendency for the ultimate pile capacity to be significantly over predicted. In order to correct for this effect, a series of soil dependant average correction factors were developed by which the derived static load multiplied to obtain a corrected UPM analysis (Paikowsky, 2004). The proposed UPM correction factor (μ) for clay of 0.65 is reported to be based upon a very limited number of cases (McVay et al., 2003). More recently it has been proposed that a much greater average correction factor in clay is required, resulting in a μ value of 0.47 (Weaver and Rollins, 2010).

The results of analysis using the various procedures are shown in Fig. 6 for the cast in situ CFA pile where the

Table 4
Soil viscosity parameters (Middendorp et al., 2008).

Soil type	Viscosity index, $I_{v\alpha}$
Sandy silt	0.018
Silt	0.025–0.032
Clayey silt	0.015–0.038
Silty clay	0.017–0.034
Clay, medium (intermediate) plasticity	0.03
Clay, high plasticity	0.04
Clay (bentonite)	0.06
Peat	0.07

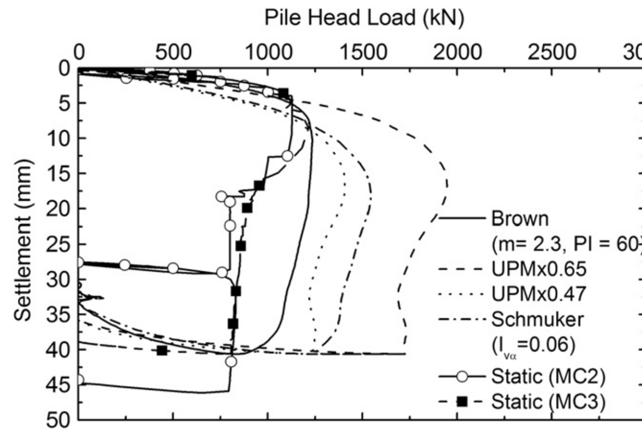

Fig. 6. Comparison of RLT analysis techniques with measured static pile resistance for a cast in situ CFA pile (R1).

viscous rate parameter m in the Brown method has been set at 2.3 based upon the encountered plasticity of the soil. For initial comparison, the results of the UPM analysis are shown corrected by both the proposed values of 0.47 and 0.65. In applying the Schmuker method, a value for the viscosity index of 0.06 (Table 4) was used, assuming that the reference to bentonite in the table suggests a clay of very high plasticity.

For the CFA pile, the approach proposed by Brown appears to give the best prediction of peak static capacity. The other approaches do not perform as well although the value of UPM correction factor of 0.47 seems more appropriate than 0.65 in this case. Optimisation of the rate parameters to suit the very high plasticity soil results in a UPM correction factor of 0.38, which is a greater correction than the values previously proposed. Similarly in the Schmuker method, the limited guidance on parameter selection would suggest values in the range 0.04–0.06, but again a larger optimised correction was required with a value of 0.082 to obtain reasonable agreement. This is outside the range of values given in Table 4. Although performance of the Brown method to predict peak capacity is encouraging, the ability of all of the analysis techniques to emulate the strain softening behaviour seen in the static tests is relatively poor.

To allow direct comparison between the analysis of the cast in situ CFA pile and the driven displacement pile identical rate effect parameters were used (as the soil is identical in each case). In contrast the results of analysis on the driven pile show significant under prediction of peak equivalent static capacity for both the Brown and Schmuker techniques (Fig. 7). The UPM approach adopting a correction factor of 0.65 performs the best with a 14% over prediction of static capacity. It should be noted that comparison is made between the results of RLT analysis and measured standard rate CRP at the settlement relating to the peak static force derived from RLT. The peak capacity predicted by the Brown method is only 65% of that measured. Again, there is little apparent strain softening suggested in the derived equivalent load-settlement

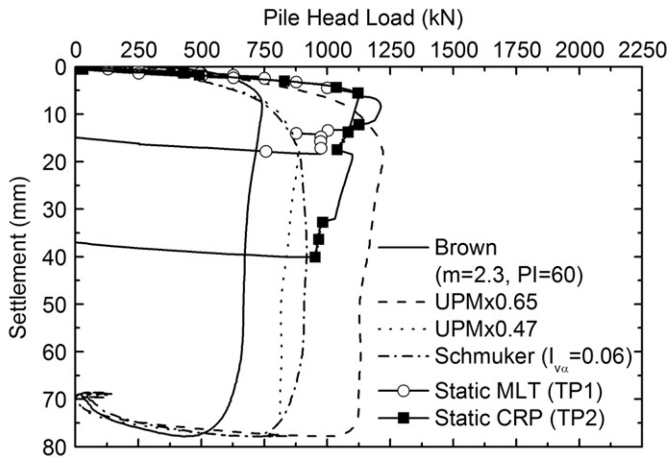

Fig. 7. Comparison of RLT analysis techniques with measured static pile resistance for a driven precast pile (S2).

response, although the strain softening measured during the static testing of the driven piles is reduced when compared to the CFA piles (Fig. 2). It should also be noted that both the UPM and Schmuker methods suggest that peak static derived capacity occurs at significantly greater settlement than that measured in static testing or that derived in the Brown analysis.

The apparent under prediction of pile capacity by the analysis techniques for driven piles is caused by the inability of the techniques to distinguish between different types of piles and installation techniques. For example, the Brown technique (Eq. (4)) was developed based upon auger bored and CFA cast in situ piles supplemented by high speed laboratory model pile and probing tests, hence the good agreement with the CFA piles in this study using default rate parameters. The Schmuker method has its origins in the low strain rate laboratory element testing of a variety of soils (Krieg and Goldscheider, 1998). The UPM technique appears to have been developed for use with a wide range of soils and pile types. On investigating the origins of the UPM correction factor μ used, it would appear that the value of 0.65 for clay is based upon a relatively low number of pile tests and sites (McVay et al., 2003, Paikowsky, 2004). It would also appear that the majority of the piles used to develop the 0.65 factor found were displacement piles and would thus explain the better performance of UPM for the driven piles in this study when a factor of 0.65 was adopted. The more recently proposed UPM correction of 0.47 (Weaver and Rollins, 2010) performs better for the CFA piles than the driven, which is again due to the correction being developed for cast in situ piles only. This pile type dependant limitation of the application of the UPM correction parameters does not appear to have been previously identified.

In clays, it is well known that driving piles significantly enhances the shaft capacity typically by 30%, with cast in situ techniques only displaying 70% of the shaft capacity obtained from a driven pile (Fleming et al., 2009). This effect was highlighted earlier in the paper by the variation in total stress adhesion factors (Fig. 2). For example, by reducing the measured static peak capacity (Stage B) of the driven pile TP2 (Fig. 7) to 70% of its measured static capacity to 795 kN, the results are well within the limits of the static prediction (from RLT analysis). Thus, the difference between the predicted equivalent static capacity of the driven pile and that measured (Fig. 7) may be assumed to be the result of the difference between the static capacities typically encountered when comparing cast in situ non-displacement piles to driven piles and not as a result of a variation in rate effects associated with differences in pile installation techniques. As noted earlier, viscous rate effect parameters were found to be unaffected by pile installation techniques when analysing the results of high rate CRP tests (CRP(H)). Thus, assuming simplistically that non-displacement piles only display 70% of the driven equivalent static capacity leads to the modification of Eq. (3) shown below for the assessment of the ultimate capacity of driven piles:

$$\tau_{\lim} = 1.3\tau_s\left[1+m\left(\frac{\Delta v}{v_0}\right)^n\right] \tag{7}$$

$$\Rightarrow \frac{\tau_{\lim}}{\tau_s} = 1.3+1.3m\left(\frac{\Delta v}{v_0}\right)^n \tag{8}$$

Such an approach allows the original database of viscous parameters to be utilised for analysis. It is acknowledged that simply increasing the static shaft capacity utilised in the analysis by 30% to reflect the enhancement due to driving is a very simplistic approach. It is also acknowledged that assessing the effects driving has on pile capacity is relatively complex and difficult to predict accurately with complex analysis techniques still relying heavily on empirical correlation (Randolph, 2003).

The results of applying Eq. (4) modified to incorporate the "30% enhancement" in the form shown in Eqs. (7) and (8) are shown in Fig. 8. What the approach appears to suggest is that the magnitude of the rate effect is relatively unaffected by the driving process and it is only the enhancement of the static pile capacity due to driving that is causing the differences in the results shown in Figs. 7 and 8. This observation is

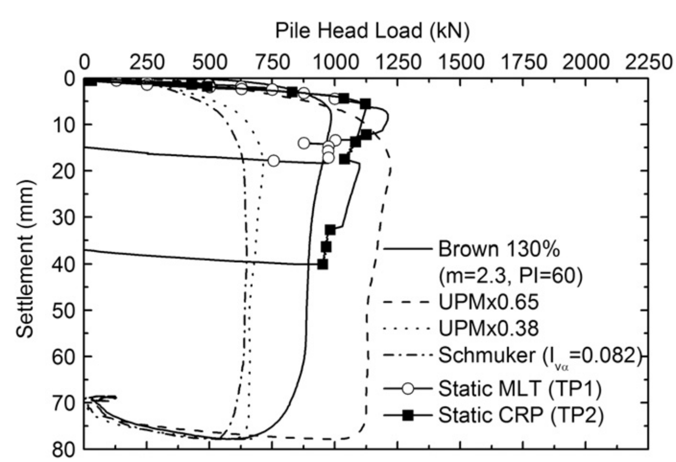

Fig. 8. Comparison of RLT analysis techniques modified to suit driven pile installation with measured static pile resistance for a driven precast pile (S2).

tentative as slight variations in the rate effect are inevitably masked by the accuracy of the "30% enhancement". Optimisation of the results suggests that the enhancement of capacity due to the pile being driven is greater than 30% and is actually better represented by a 35% enhancement. To highlight the improvement to the Brown technique, the UPM and Schmuker methods are shown with correction factors optimised to suit the very high plasticity clay based upon the CFA testing results (Fig. 6), but not the "30% enhancement" (Fig. 8).

As previously mentioned, the UPM correction factor of 0.65 works well for the driven piles (Fig. 8) with optimisation in the high plasticity London Clay, giving a value closer to 0.62. Reduction of this optimised value to 65% of its original magnitude (i.e. assuming 35% increase in static pile capacity for driven piles) suggests a correction factor μ for a cast in situ pile of 0.40, which is close to 0.38 derived for cast in situ testing in the very high plasticity clay (Brown and Powell, in press). Again, this highlights the need for the UPM analysis to take into account the method of pile installation, but by adjusting the existing parameters, it may be possible to simply estimate a correction factor appropriate for various pile installation techniques.

Similarly, the viscosity index proposed by Schmuker reduces from 0.082 to 0.054 to suit the analysis for driven piles. This new viscosity index value for the driven pile is closer to values recommended for high plasticity clay (0.04) and organic clays and bentonite (0.06) (Krieg and Goldscheider, 1998), all of which are assumed to be similar to the very high plasticity soils encountered at this site. The Schmuker viscosity index values have previously been criticised for being too low when selected based upon soil type (Brown and Powell, in press). This has been attributed to the relatively low velocities used in the laboratory tests when deriving the parameters. The reduced viscosity index value of 0.054 obtained for driven piles above appears to fit with the parameters proposed by Schmuker, but this is thought to be purely coincidental based upon the laboratory origins of the method.

Thus, rather than suggesting that the published parameters for the various RLT analysis techniques are appropriate for all pile types, it seems more appropriate to use them for the specific pile types and the installation methods from which they are originated. For example, when testing in fine grained soils, the current UPM and Schmuker correction parameters are more appropriate for driven or displacement piles, and those proposed for the Brown method seem to work for cast in situ or non-displacement piles. Therefore, further investigation in to the analysis of RLT tests in fine grained soils must distinguish between different pile and installation techniques and be based upon case study information or testing that accurately models pile installation.

5. Conclusions and recommendations

Based upon this study, it would seem appropriate that the analysis of RLT must acknowledge the type of pile installation being tested. For the RLT and static CRP tests presented, it would seem that there is no discernible difference between the rate effects experienced in the RLT testing of driven precast piles and cast in situ piles. The differences in RLT analysis performance observed seem to be the result of the enhanced static pile capacity often associated with the installation of driven piles in clays. As current analysis techniques in the majority are based upon empirical correlation with static pile tests, in the future developments and applications of RLT analysis, it is important that the potential difference in static capacity that may occur for different pile installation methods in different soils is acknowledged.

Existing UPM correction parameters for clays appear to have their basis predominantly in the testing of driven piles and should be applied to other pile types with caution. Ideally, new correction factors appropriate to the particular pile installation technique should be derived. In the absence of these, it may be appropriate to increase the effect of the UPM correction factor to reflect the reduced static capacity associated with cast in situ piles. A similar approach may also be used to modify the analysis proposed by Brown and Hyde (2008), which would allow the use of existing soil specific rate parameters. In both cases, this requires the ability to derive the difference between driven and cast in situ static pile capacity prior to testing, and this is far from straightforward. The Schmuker method also appears to require further development to derive appropriate rate correction factors suitable for RLT.

At the current level of understanding of RLT analysis, it would seem appropriate to recommend that where RLT is specified, there should be documented experience of testing and analysis in both that soil type and for the pile type and installation method proposed. This recommendation seems appropriate until there is greater documented experience of RLT use for a wide range of soil and pile/installation types.

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